Proceedings of the Pakistan Academy of Sciences:
A. Physical and Computational Sciences 55 (4):45–58 (2018)

ISSN: 2518-4245 (print), 2518-4253 (online)

Pakistan Academy of Sciences

Resarch Article

Effect of Adding High Strength Concrete Topping on Flexural and Shear Behavior of Hollow Core Pre-stressed Slabs

Liaqat Ali Qureshi[1], Ali Ajwad[2*], Muhammad Ali Khan[3], Gouhar Sultana[3], Muhammad Adil Khan[3], Asjad Javed[3], Ali Aqdas[3], and Humaira Kanwal[3]

[1]Department of Civil Engineering, University of Engineering and Technology Taxila, Pakistan
[2]Department of Civil Engineering, University of Management and Technology Lahore, Pakistan
[3]Department of Civil Engineering, University of Lahore, Pakistan

Abstract: This paper presents an experimental study regarding the effect of adding extra topping on flexural& shear capacity of hollow core pre-stressed concrete slabsand mainly focuses on how to increase shear and flexural capacity of slabs on site with the help of in-situ concrete topping. There may arise a situation where a depth greater than 200 mm and less than 265 mm may be required due to particular applied loading. Such extra depths can only be produced by applying in-situ concrete topping layer. High strength concrete toppings of 32 mm and 65 mm thicknesses were used on slabs of 200 mm original depth and were also compared with slab of 265mm depth. Out of four topped slabs tested in the program, two were tested under flexure and the other two under shear loading. Two control slabs of 200 and 265 mm but without extra topping were also tested in the same way. The ultimate load capacity, shear capacity, deflection, crack pattern, crack width and modes of failure were investigated for each slab. From the findings, it was concluded that the addition of concrete topping could enhance the flexural and shear capacity of hollow core units. For flexural loads, the increase was up to sixty percent for 65 mm topping and twenty-three percent for 32 mm topping. For shear testing, the increase was almost hundred percent for 65 mm topping and fifty-six percent for 32 mm topping. It was also observed that both flexural and shear strength of topped slabs was almost equal to the strength of un-topped slabs of same original depth. In cost comparison, the topped up slab came out to be 8.45% expensive as compared to hollow-core slab of similar depth

Keywords: Hollow core pre-stressed slab, Topping, Cracking, Deflection, Shear strength, Flexural strength

1. INTRODUCTION

Concrete is one of the most widely used, versatile and economical material in today's construction industry. Concrete as a structural component can be seen in buildings and bridges in various forms. For the development of an overall efficient and safe structure, it is required to understand the response of these forms under different types of loading conditions [1]. Although concrete is being used as a primary material since early 1960s, the form in which it is used has been changing with time. Pre-stressed hollow core slabs are the most common form of precast flooring used world-wide, with an annual production of about 20 million cubic meters in Europe. These represent about 40 to 60% of the overall precast flooring market [2] and are mainly used in the construction of car parking, industrial and residential buildings and sports stadiums. A typical hollow core slab is a monolithic pre-stressed or reinforced concrete unit with the overall depth divided into upper and lower flanges linked with vertical webs, constituting longitudinal holes, the cross section of which remains constant throughout. Scott presented a load test on hollow core slabs with concrete topping and came up with an answer that the composite action between the slab unit and additional topping was observed until the failure load [3]. Bayasiand Kaisar [4] used RCC topping but results showed that certain number of steel studs was needed to reduce failure potential, which was obviously the result of inadequate stress transfer.

Received: May 2018; Accepted: December 2018
* Corresponding Author: Ali Ajwad; ali.ajwad@umt.edu.pk

Flexural behaviour of hollow core slab units with concrete topping was also studied by Dowell and Smith [5] who clarified that the topping acted satisfactory in flexure and there was no shear slip at the interface of original slab and additional topping during the application of load. Rehman et al.[6] did research on hollow core slabs of different depths and concluded that for certain span lengths and prestressing tendons, the failure mode changed from pure flexure to flexure-shear for more than 200 mm deep slabs. Silfwerbrand [7] studied the bond between old and fresh concrete and concluded that the bond strength with proper workmanship and treatment of the interface was 3 MPa.

Micallef [8] performed research to assess the shear capacity of pre-stressed hollow core floor units. The hollow core slabs were tested and analyzed under the application of a knife edge load at a distance of 575mm from the support. The failure load was then compared with the respective safe calculated and load deflection curves were drawn to assess the mode of failure. Brooand Lundgren [9] presented a detailed analytical study of finite element analysis on hollow core pre-stressed concrete slabs subjected to shear and torsion. He found out that shear failure starts with a bending crack that turns into an inclined crack and ends with a shear displacement along the crack. Hawkins and Ghosh [10] concentrated on the shear strength of hollow-core slabs andobserved that if bond slip of the tendon occurred, it was only after the formation of the shear crack. In some of the test specimens, it was also noted that bond slip of the tendons was not the cause of shear failure. Girhammer and Pajari[11] studied the effect of additional concrete topping on the shear capacity of hollow core slab units. They found that the bond at the interface of original slab and additional topping adequate and noted the increase in shear capacity to be around 35 percent. Ibrahim et al. [12] did an experimental study on the shear behaviour of precast concrete hollow core slabs with concrete topping and concluded that the hollow core unit surface condition and longitudinal joint affect the stiffness and shear-flexure strength of the slabs. The optimum hollow core unit surface condition which can produce highest stiffness and shear strength is rough and wet conditions, while the longitudinal joint between hollow core unit panels reduces the slab shear strength. Eom et al. [13] did a research on evaluation of Shear Strength of Non-prestressed Reinforced Concrete Hollow-Core Slabs and concluded that the shear strength of hollow-core slab was degraded as the void ratio increased but hardly affected by other factors including the effective width of web. Pachalla and Prakash [14]studied Load resistance and failure modes of glass fiber reinforced polymer (GFRP) composite strengthened hollow core slabs with openings and concluded that GFRP strengthening is an effective and cost viable technique for restoring the strength and stiffness of precast prestressed hollow core slabs due to openings. Hwang et al. [15] did research on flexural capacity of Precast Concrete Triple Ribs Slab (TRS) and their results revealed that TRS had enough flexural strength and ductility to resist the design loads and its strength can be suitably predicted by using code equations.

Usually, at the time of production, surface of hollow core slab units is not a level and smooth surface. Instead, it is turned smooth and level at the site using screed or concrete topping. The thickness of such screed or topping is usually too small to have any effect on the flexural capacity of the slab units. In Pakistan hollow core slabs are available in two standard depths i.e 200 mm and 265 mm. There may arise a situation where a depth greater than 200 mm and less than 265 mm may be required due to particular applied loading. Such extra depths can only be produced by applying in-situ concrete topping layer. All the existing research tends to increase capacity of slab units by increasing slab depth at the time of manufacturing whereas research focuses on how to increase shear and flexural capacity of slabs on site with the help of in-situ concrete topping. However, the bond between old and fresh concrete should be sound. In this research, effect of additional topping on flexural strength of 200 mm depth slabs has been investigated. The results obtained were compared with corresponding hollow core slabs of 200 mm and 265 mm original depths.

2. EXPERIMENTAL PROGRAM

2.1. Testing Arrangement

The test specimens consisted of a total number of six slabs. Two were control slabs abbreviated as CONT200, CONT265 having 200 mm and 265 mm depth respectively. The rest of all four slabs

Table 1. Description of test slabs

Sr. No.	Slab	No. of Slabs	Slab depth (mm)	Topping depth (mm)	Topping type
1	CONT200	1	200	-	-
2	CONT265	1	265	-	-
3	HST32	2	200	32	High strength
4	HST65	2	200	65	High strength

were of 200 mm original depth. Two of these slaps were topped with 32 mm and the other two with 65 mm high strength concrete layer. The control slab of 200 mm depth (CONT200) was tested to study the effect of changing additional topping depth on its flexural and shear strengths. The control slab of 265 mm depth i.e., CONT265 was tested to compare its flexural and shear behaviour with 65 mm topped slabs with the same total depth of 265 mm. The surface of the slabs was roughened with steel brush before the application of topping for all the specimensto create a good bond between the old and the new concrete. Table 1 summarizes the detailed description of slab specimens tested in research program.

Hollow core slabs had a width of 990 mm with five 155 mm diameter holes running along 2400 mm length of slab. The pre-stressed strands of 10 mm diameter were used and an initial induced pre-stress was of 110 MPa. The area of steel used was 235.65 mm^2 in both CONT200 and CONT265. The cross sections of 200 mm and 265 mm slabs are shown in Figures 1 and 2 respectively whereas the testing arrangement for the application of load in flexural loading and shear loading are shown in Figures 3 and 4 respectively.

Tests were performed after 28 days of the application of topping. The hollow core pre-stressed slabs were tested under flexural and shear test conditions. Schematic diagrams of flexural and shear testsare shown in Figures 5 and 6. The load increment of 10 kN applied by a hydraulic jack was selected until the failure of slabs. Dial gauges were fixed on underside of the slab right below the point of application of load for the measurement of deflection. The development of first and subsequent

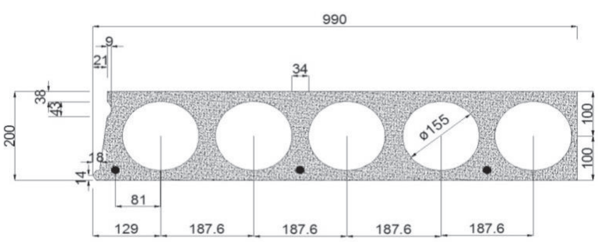


Fig. 1. Cross section of hollow core slab with 200 mm depth

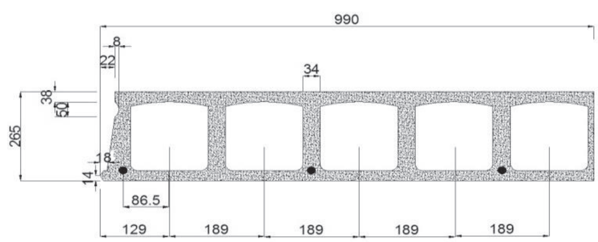


Fig. 2. Cross section of hollow core slab with 265 mm depth

Fig. 3. Testing arrangement for slabs in flexural loading

Fig. 4. Testing arrangement for slabs in shear loading

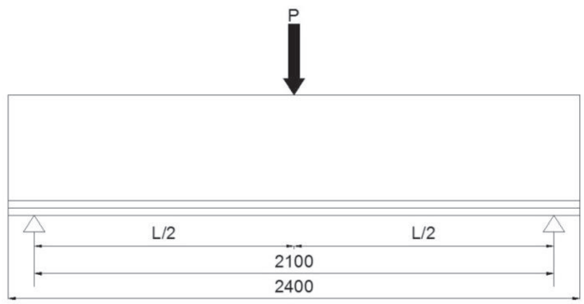


Fig. 5. Schematic diagram of flexural test

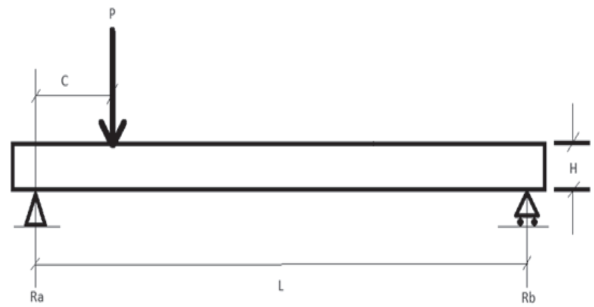


Fig. 6. Schematic diagram of shear test

cracks appeared up to failure of slabs was carefully monitored and the respective loads were noted.

2.2. Compressive Strengths of the Toppings

Three cylinders and three cubes were cast for each mix of concrete toppings and their compressive strength tested at 28 days age. Table 2 shows the average compressive strength values for 32 mm and 65 mm topped concretes. The compressive strength test was performed in accordance with the ACI code ASTM C42/C42-M-04 for cylinders and British Code BS EN 12390 for cubes. However, the 28 days compressive strength of concrete used in the hollow core slabs was 50N/mm^{2}as provided by the manufacturer.

Table 2. 28 days Compressive Strength of topping concrete

Sr. No.	Specimen	28 day's cylindrical strength (N/mm^2)				28 day's cubical strength (N/mm^2)			
		1	2	3	**Average**	1	2	3	**Average**
1	HST32	48.5	49.1	48.8	48.8	53	53.3	53.3	53.2
2	HST65	53.6	53.2	52.2	53	55	58.4	55.6	56.3

2.3. Bond Strength at the Interface

In order to check the bond strength at interface of old and fresh concrete, 3 cores were cut from each topped slab and tested for direct tensile strength in the Universal Testing Machine. According to Silfwerbrand, the tensile bond strength varies between 1 to 2 MPa for normal strength concrete [7] but in this case the tensile bond strength came out to be more than 2 MPa for all specimens which proved that the bond between the old and the new concrete performed well during testing. Table 3 presents the strength of cores and Figure 7 shows the testing setup used for direct tensile strength test in Universal testing machine.

Table 3. Tensile strength of cores

Sr. No.	Specimen	Tensile strength (N/mm^2)			
		1	2	3	Average
1	HST32	2.1	2.3	2.2	2.2
2	HST65	2.05	2.2	2.2	2.15

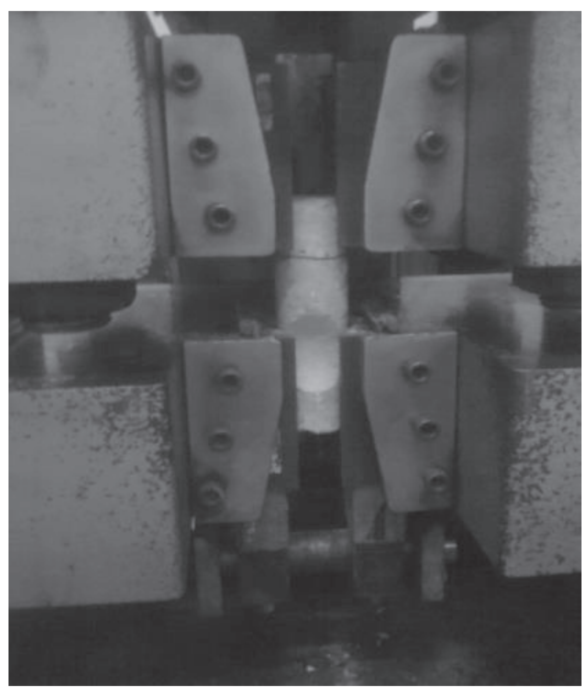


Fig. 7. Direct tensile strength test on composite slab cores

3. RESULTS AND DISCUSSION

3.1. Flexural Capacity

3.1.1. Ultimate Moment

Experimental ultimate moments were calculated as a product of failure load noted during flexural testing and its perpendicular distance from the support. Theoretical ultimate moment capacity was also calculated using the formula given by Elliot [16]:

$$M_u = f_{pb} A_{ps}(d - d_n) \quad \text{(eq.1)}$$

M_u = Theoretical ultimate moment capacity (kNm)

f_{pu} = Ultimate tensile strength in tendons = 1770 N/mm^2

f_{pb} = Design stress in tendons = $0.87 f_{pu}$ = 1540 N/mm^2

A_{ps} = Area of prestressing steel tendons = 235.65 mm^2

d = y^t(distance to centroid) + e(eccentricity) mm

d_n = Depth of compression zone

$= 2.47\{[A_{ps} f_{pu}/(b d f_{cu})][f_{pb}/f_{pu}]d\}$

$= 2.47\{[(235.65 \text{ x } 1770/(990 \text{ x } 200 \text{ x } 50)]\ [1540/1770]200\} =$ 18 mm

b = Breath of Slab = 990 mm

d = Depth of slab = 200 mm

f_{cu} = Compressive strength of hollow core slab concrete = 50 N/mm^2

Table 4. Experimentation results for all slabs tested

Specimen	Cracking Moment (kNm)		Maximum Midspan Deflection (mm)		Failure loads for slabs tested under flexural loading (kN)			Failure loads for slabs tested under shear loading (kN)		
	Theo-retical	Experi-mental	Theo-retical	Experi-mental	Load at first crack	Yielding point	Breaking point	Load at first crack	Yielding point	Breaking point
CONT200	48.96	52.25	57	61	75	85	100	110	118	140
HST32	64.11	63.08	63	59	90	102	112	170	180	213
HST65	87.37	84.1	94	93	120	131	139	225	232	260
CONT265	87.5	88.1	99	107	125	135	140	228	237	258

Table 4 shows a comparison between theoretical and experimental ultimate moment capacities for the slabs under flexural loading. Comparison shows that experimental results of ultimate moment capacity are well in accordance with the theoretical values calculated by using equation 1.Comparison of topped slab HST32 with control slab CONT200 shows that there was an increase in ultimate moment capacity when 32 mm topping layer was applied over 200 mm thick slab. Also there was considerable increase in moment capacity in case of HST65 when 65 mm thick topping layer was applied over 200 mm thick control slab. Furthermore, comparison of topped slab HST65 with control slab CONT265 proves that ultimate moment capacity is in the same range for both the cases.

3.1.2. Cracking

During the application of flexural loading on pre-stressed hollow core slabs, cracks appeared near mid-span as shown in figure 8. The cracks started from the bottom of the slabs and propagated vertically upward. The cracks appeared gradually with the increase of load and when these reached in the upper half of slabs, these became diagonal and travelled towards the point of application of load. The change in direction occurred abruptly and slabs failed completely when the cracks reached at the top of slabs. For theoretical calculation of cracking moments, the following equation was used as mentioned by Foubert [17].:

Table 4 shows the comparison of theoretical and experimental cracking moments for all the slabs. Again the experimentally observed and theoretically calculated cracking moments are found having nearly same values. Cracking moments behaved in a similar pattern as noted in case of ultimate moment for the tested slabs.

3.1.3. Cracking at the Interface of Slab and Topping

After failure, cracks appeared at the interface of slabs and additional toppings for both the HST32 and HST65. This shows that topping interacts monolithically with the slab and no shear slip occurred during the application of load similar to thefindings of Dowell and Smith [5].

3.1.4. Deflection

During testing, deflections were measured using dial gauges placed under the slabs right below the point of application of load. For calculating the theoretical values of deflection, (eq.3) derived by Bhatt, was used [18]. This equation is valid for flexural loading only because it takes into consideration the maximum bending moment that occurs at the mid-

(a) Crack patterns of 265mm depth slab

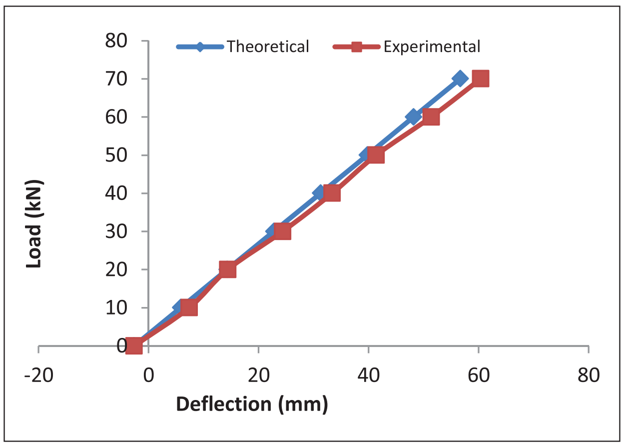
(b) Crack patterns of 200mm depth slab with 32mm topping

(c) Crack patterns of 200mm depth slab with 65mm topping on top

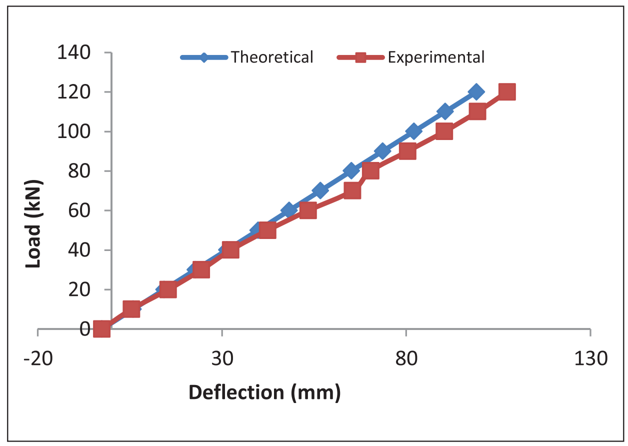
Fig. 8. Crack pattern of slabs under flexural loading

$$M_{cr} = \left[\frac{P_e}{A_g} + \frac{(P_e)(e)(y_b)}{I_g} + 0.6\lambda\sqrt{f'_c}\right]\frac{I_g}{y_b} \qquad \text{(eq.2)}$$

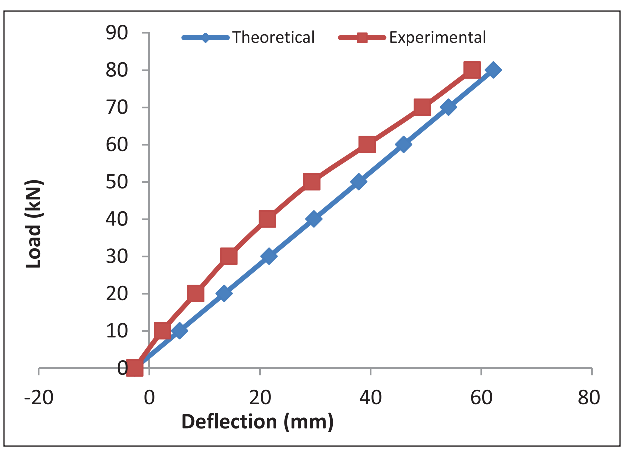
M_{cr} = Cracking moment
P_e = Effective prestressing force after applicable losses
A_g = Gross area of the concrete section (including reinforcement)
e = Eccentricity from the neutral axis to the internal reinforcement
y_b = distance from the neutral axis to the bottom fiber in tension
I_g = gross moment of inertia
λ = density factor of concrete
f_c' = Specified concrete strength



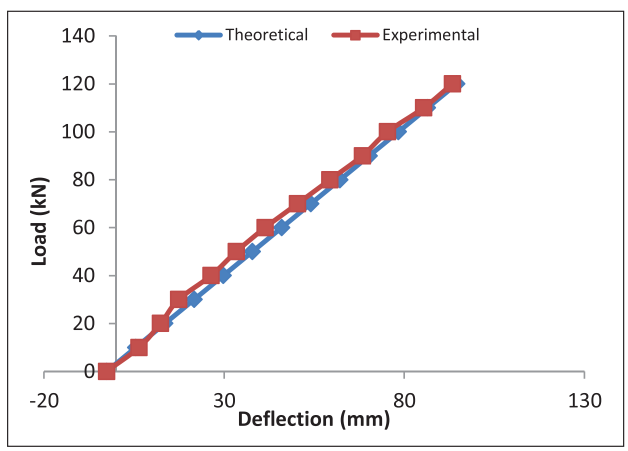
(a) Load-deflection curves for CONT200 up to cracking under flexural loading



(b) Load-deflection curves for CONT265 up to cracking under flexural loading



(c) Load-deflection curves for HST32 up to cracking under flexural loading



(d) Load-deflection curves for HST65 up to cracking under flexural loading

Fig. 9. Load-deflection curves for slabs tested

span of slabs. Topping was considered as a part of slab when calculating moment of inertia. Also, this equation was only applicable before the appearance of first crack.

Figure 9 gives comparison between experimental and theoretical values of net deflection. In these figures, the theoretical values were obtained from equation 3 and 4 whereas experimental values

$a_e = (L^2/E_{c,t}I)\,\Sigma\, k\, M$ eq. (3)

a_e = Deflection (mm)

L = Span of slab = 2400 mm

M = Maximum bending moment in the span (N-mm)

k = Factor depending on the shape of bending moment diagram.

I = Moment of inertia (mm^4)

$E_{c,t}$ = Modulus of elasticity at an age t (days) derived from the following equations:

$E_{c,20} = 20 + 0.2 f_{cu,28}$

$E_{c,t} = E_{c,20}(0.4 + 0.6\, f_{cu,t}/f_{cu,28})$ Where $t > 3$ days

For calculating camber due to pre-stressing, the following equation was used:

$a'_e = L'^2/(E_{c,3}I)\,(1/8)(-\eta_t P_{op} e_p)$ eq.(4)

Where:

a_e' = Camber (mm)

L' = Span of slab between supports = 2100 mm

K = Factor that depends on the shape of bending-moment diagram

I = Moment of inertia (mm^4)

$E_{c,3}$ = Modulus of elasticity at an age of 3 days which can be derived from the following equations:

$E_{c,20} = 20 + 0.2 f_{cu,28}$

$E_{c,t} = E_{c,20}(0.4 + 0.6\, f_{cu,3}/f_{cu,28})$

$-\eta_t$ = Constant equal to -0.89

P_{op} = Initial prestressing force = 110 $\frac{N}{mm^2}$

e_p = Eccentricity of tendons = 68.4 mm

By substituting all the values, camber can be calculated as:

$$a'_e = L'^2/(E_{c,3}I)\Sigma kM = L'^2/(E_{c,3}I)\,(1/8)(-\eta_t P_{op} e_p)$$
$$= 2100^2/(22.8 \times 10^3 \times 659.01 \times 10^6) \times (1/8) \times (-0.89 \times 235.65 \times 10^3 \times 68.4)$$
$$= -0.53 \text{ mm (upwards)}$$

were taken from testing. Net deflection means experimental/theoretical deflection minus camber due to pre-stressing.

It can be seen from Figure 9 that for all slabs, experimental and theoretical values of deflections were almost in the same range until the appearance of first crack. Also, the experimental deflections of topped slabs i.e., HST32 &HST65 came out to be less than theoretical values whereas in case of CONT200 and CONT265 the situation was different. This was due to the fact that the topping acted as part of slab which increased the distance'd' i.e., sum of the distance to centroid and eccentricity resulting in greater moment capacity. Also, when slabs of similar depths were compared, the topped slab, HST65 showed less deflection than that of CONT265. Table 4 shows comparison of all slabs and it can be seen that the topping would result in 15 to 20 percent less deflection as compared to un-topped slabs. Same sort of behavior was seen while comparing the theoretical values.

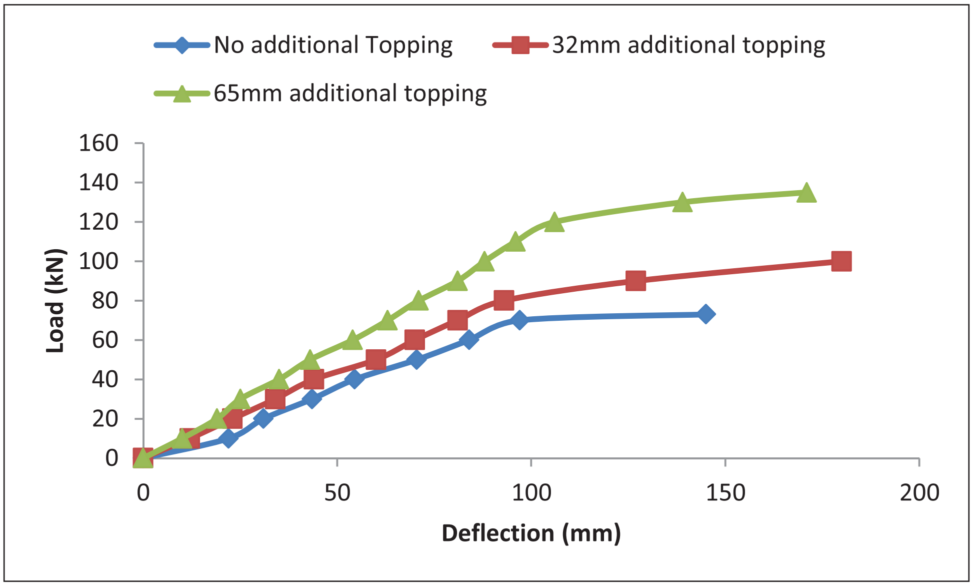


Fig. 10. Load-deflection relationship for high strength topping over 200 mm slab under flexural loading

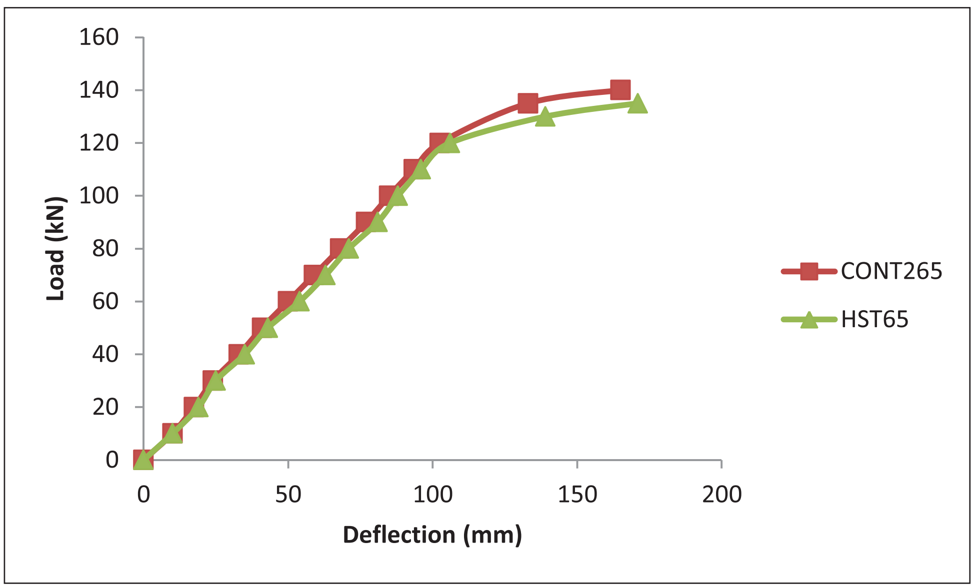


Fig. 11. Load-deflection relationship for CONT265 and HST65 under flexural loading

3.1.5. Effect of Topping Depth

Figure 10 shows load-deflection curves for 200 mm slab without topping and with toppings. It is evident from the figure that flexural capacity increased up to 60 percent with the increase in topping depth. When concrete failed and load transferred to the pre-stressed strands, deflection increased linearly with the increase in load. All slabs failed within 15 kN increase in load after first crack appeared. Also, in all slabs, vertical cracks were noticed initiating from the bottom of slabs directly under the point of application of load and propagating towards the top. This shows that all slabs failed under pure flexure.

A comparison of the first cracking, yielding point and breaking point loads for all slabs can be seen in Table 4. It can be noticed from the Figure that increase in flexural capacity is 23 percent for 32 mm topping and 60 percent for 65 mm topping as compared to 200 mm slab without topping. Also, slab of 200 mm depth with 65 mm topping (HST65) achieved flexural strength almost equal to

CONT265, with same total depth.

Figure 11 shows comparison of load-deflection curves for both slabs with the same total depth that is of 265mm. It shows that flexural capacity for two slabs was almost the same i.e 120 kN and corresponding displacements were 98 mm for HST65 and 105 mm for CONT265. It means that whether a precast slab of 265 mm depth is used or a 200 mm depth slab with 65mm topping is used, the total load they would carry will be the same. It is also proved that topping interacted monolithically with the slab and no shear slip occurred during the application of load due to proper roughening of the main slab surface at the interface with topping.

3.2. Shear Capacity

3.2.1. Cracking in the Slab Units

In case of shear loading, hairline cracking started to appear near supports and sudden failure occurred when cracks propagated y at an angle of 40 degree towards the point of application of load. After failure, a little increment in loading resulted in prominent cracks along the cores and web of slabs. Failure patterns can be seen in Figure 12.

3.2.2 Cracking at the Interface of Slab and Topping

After failure, cracks appeared at the interface of slabs and additional toppings for both HST32 and HST65. This shows that topping interacts monolithically with the slab and no shear slip occurred during the application of shear loading.

3.2.3. Deflection

In order to check the deflection of slabs under shear loading, dial gauges were installed under the slabs, right below the point of application of load. Figure 13 shows the load-mid-span deflection relationship between control and topped slabs. It can be noticed from the Figure that deflections under application

(a) Crack patterns under shear loading for 200mm depth slab from side

(b) Crack patterns under shear loading for 265mm depth slab from side

(c) Crack patterns under shear loading for 200mm depth slab from front

Fig. 12. Crack pattern under shear loading

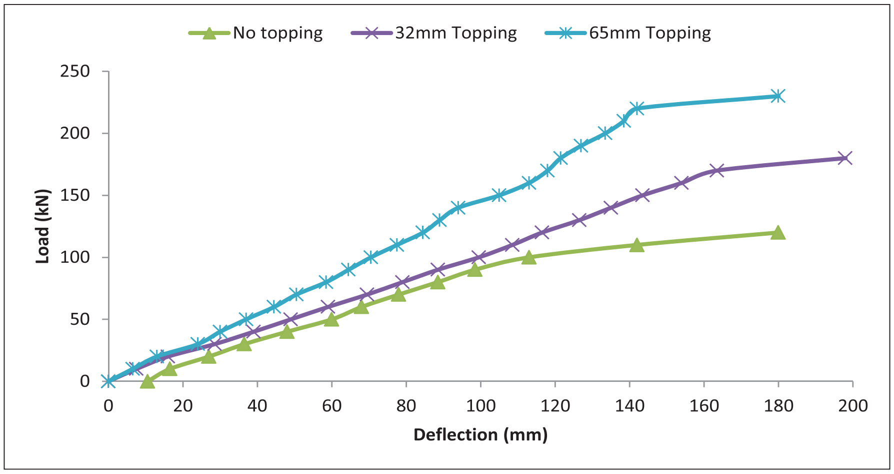


Fig. 13. Load-deflection relationship for high strength topping over 200 mm slab under shear loading

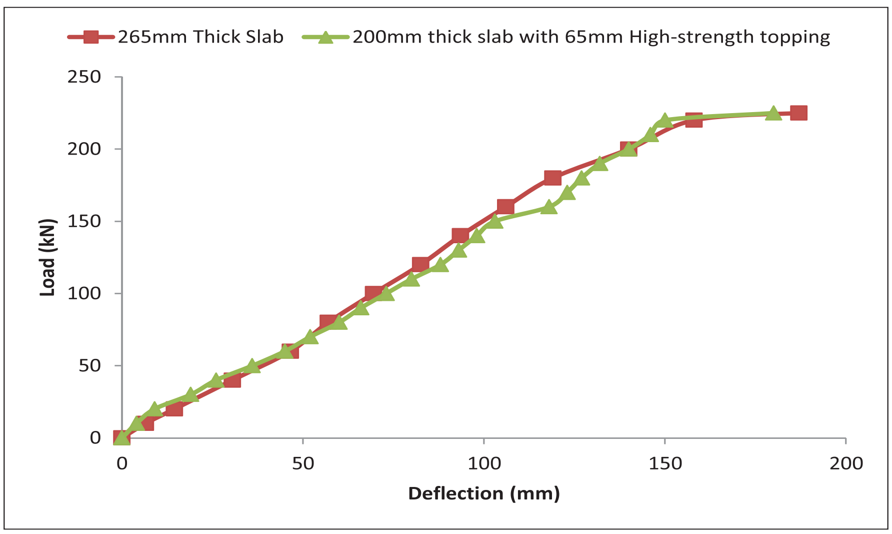


Fig. 14. Load-deflection relationship for CONT265 and HST65 under shear loading

of load were similar for all three slabs. However, the deflection in case of topped slab HST32 was found greater as compared to the other two slabs.

3.2.4. Effect of Topping Depth

Figure 13 shows load versus mid-span deflection curves for 200 mm slab without topping and with toppings. It is evident from the results that additional topping improved the shear strength of slab units linearly with the increase in topping depth. After failure of concrete, when load was transferred to the pre-stressed strands, the increase in strain was also linear with the increase in load for all specimens tested. During the course of loading, cracks appeared near the support and then propagated towards the point of application of load. It was noticed that all the slabs, with or without topping, reached their yielding point within 5 to 10 kN increase in load after the appearance of first crack which indicated an abrupt mode of failure. Almost all the cracks started to develop in between 35 to 40 degrees with the horizontalwhich is similar to the findings ofGirhammer and Pajari [11].

Different loads at which first crack appeared, as well as loads indicating yielding point and ultimate breaking point for the slabs can be seen in table 4. It can be noticed that increase in shear strength is linear with the increase in topping depth and increase in shear capacity is about 50 percent for 32 mm topping and capacity was more than doubled in case of 65 mm topping as compared to 200 mm depth slab with no topping i.e. control slab.

When compared with the control slab of 265mm depth, the failure load for the 65 mm high-strength concrete topped slab was almost the same as shown in Figure 14. This shows that whether a precast slab of 265mm depth is used or a 200mm depth slab with 65mm additional topping of high-strength concrete, the total load taken will be almost same. It was also noticed that additional topping interacted monolithically with the slab and no shear slip occurred during the application of load.

$$V_{co} = 0.67 b_v h \sqrt{(f_t^2 + 0.8 f_{cp} f_t} \qquad \text{eq. (5)}$$

Where:

V_{co} = Ultimate shear capacity (kN)

b_v = Width of web (mm)

h = Depth of slab (mm)

f_t = Maximum principal tensile stress (N/mm^2)

f_{cp} = Compressive stress due to prestress (N/mm^2)

3.2.4. Comparison with Theoretical Values

For comparison of the values obtained from experimental testing, theoretical values were calculated using the formula derived by Elliot [16]. Additional topping was added to the depth of slab when calculating the ultimate shear capacity. The following equation was used:

Figure 1 to 14 shows the theoretical and experimental ultimate shear capacity values for ultimate loads in the slabs during application of the load. It can be seen from the Figure that most of the experimental values came out to be similar as compared with the theoretical values.

4. COST COMPARISON

The second main aspect that governs civil engineering projects is the economy after safety considerations. That is why, a cost comparison of factory manufactured control slab with that of topped slab of same depth is carried out.

The slabs were bought from the manufacturer at a rate of PKR 7592 per cubic meter. Total cost of

Volume of CONT265	=	(2.44 x 1 x 0.265) ft^3
	=	0.6466 m^3
Cost of CONT265	=	PKR 7592 x 0.6466
	=	PKR 4908
Volume of CONT200	=	(2.44 x 1 x 0.2) ft^3
	=	0.488 m^3
Cost of CONT200	=	PKR 7592 x 0.488
	=	PKR 3705
Cost of additional 65 mm topping		
Amount of cement used	=	150 kg
Cost of cement	=	PKR 10.2 x 150
	=	PKR 1530
Cost of sand	=	PKR 1.25 x 0.8 x 14.4
	=	PKR 14.4
Cost of crush	=	PKR 1.25 x 2.2 x 25.6
	=	PKR 70.4
Cost of HST65	=	PKR 3705 + 1530 + 14.4 + 70.4
	=	PKR 5319.8
Difference in cost	=	PKR 5319.8 – 4908
	=	PKR 411.8

control slab can be calculated as follows:

It can be seen that the cost of topped slabs comes out to be 8.45% more as compared to the factory manufactured slab of same depth.

5. CONCLUSIONS

1. High strength concrete topping can be used to increase the flexural and shear capacities of hollow core slab units. It is seen that up to 60 % increase in flexural capacity can be achieved with 65 mm topping and around 23 % with 32 mm topping. Similarly, increase in shear capacity is about 50 %for 32 mm topping and capacity is more than100% in case of 65 mm topping as compared to 200 mm depth slab with no topping i.e. control slab.
2. The increase in shear and flexural capacities of topped up slabs can be considered as linear with increase of topping depth. Thus, in a situation where the flexural or shear capacity required is greater than that of 200mm depth slab and less that than of 265mm depth slab, in-situ topping of the exact depth can be used to save cost.
3. The flexural and shear capacities of the concrete topped slabs were found equal to the capacities of the control slab of similar depth without topping.
4. Experimental and theoretical moment capacities fit well with each other.
5. Experimental mid-span deflection versus load curve in flexural testing coincides well with the corresponding theoretical curve which verifies Bhatt's outcomes [16].
6. The increase in flexural and shear capacities of the hollow core slab units was found linear with the increase in topping depth.
7. Under shear loading, cracks appeared at an angle of 30 to 40 degrees with the horizontal from the support to the point of application of load. On the other hand, during flexural testing, the cracks started from the bottom of the slabs and propagated vertically upward and when they reached in the upper half of slabs, they became diagonal and travelled towards the point of application of the load.
8. The cost of topped slabs comes out to be slightly more as compared to factory-manufactured slab of same depth. The increase in cost is 8.45 % compared to the price of factory manufactured control slab.
9. The bond between old and fresh concrete created by roughening the surface at interface, was found satisfactory and topping interacted monolithically with the slab, as no shear slip was noticed during the application of the load.

6. RECOMMENDATION

In Pakistan, hollow core slabs are available in two standard depths i.e 200 mm and 265 mm. There may arise a situation where a depth greater than 200 mm and less than 265 mm may be required due to particular applied loading. It is recommended that if the surface is prepared well and the bond between hollow core slab and in-situ concrete topping is satisfactory, concrete topping of varying depths can be used to enhance the shear and flexural capacity of hollow core slab units.

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